THE EFFECT OF SIZE AND THERMAL EXPANSION OF AGGREGATES ON THE DURABILITY OF CONCRETE

by

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INTRODUCTION

In recent years many investigators have had the belief that much of the deterioration that occurs in concrete is due to the difference in thermal expansion of the aggregate and cement paste, especially where excessive freezing and thawing exist. Throughout this text thermal incompatibility will refer to a difference between the thermal expansion of the aggregate and cement paste.

One of the difficulties encountered in making a study of the thermal effects of aggregates, has been the lack of satisfactory methods for measuring the thermal coefficient of expansion. The Corps of Army Engineers has made use of the SR-4 electric strain gage in measuring the thermal expansion of coarse aggregates with excellent results. Various investigators have attempted to measure the thermal expansion of fine aggregates by using a dilatometer, but this method has proved difficult and the results are questionable.

The author has made a theoretical analysis of the stresses that might result in a matrix surrounding a spherical body, due to the thermal incompatibility of the matrix and a spherical body. This study indicates that the magnitude of the stresses depends upon the thermal expansion, Poisson's ratio, Young's modulus, and the size of the inclusion. It is indicated that the stress from small inclusions would be much less than that from large inclusions. If this is true then the thermal incompatibility of the mortar and coarse aggregate may have a considerable effect on the durability of concrete while that of the fine aggregate may have but very little effect.

PURPOSE

The purpose of this research is to determine the effect of the thermal expansion and size of coarse aggregates upon the durability of concrete.

GENERAL PLAN

This study has been divided into three parts as indicated below.

Part I. Theoretical analysis.

Part II. Methods used in measuring the thermal expansion of concrete mortar and coarse aggregates.

Part III. The effect of size and thermal expansion of coarse aggregates on the durability of concrete as determined by freezing and thawing.

PART I. THEORETICAL ANALYSIS

Little is known of the stresses created at the boundary of an inclusion of one material embedded in a matrix of another material. An attempt is made in this text to determine the probable theoretical stresses in a concrete mortar surrounding a spherical inclusion of aggregate. In deriving an expression for the tensile stresses in terms of the size, moduli of elasticity, and Poisson's ratio for the matrix and aggregate, the following assumptions are made:

1. The mortar and aggregate obey Hooke's law.

2. Each particle of aggregate may be treated as though it were a small sphere at the center of a larger sphere of homogeneous material.

Referring to Fig. 1, the spherical radii of the embedded aggregate and surrounding mortar are a and b respectively.

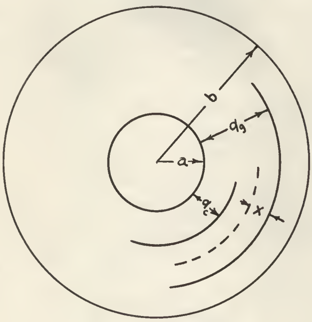

Fig. 1

The unrestrained radial displacement of the aggregate and mortar due to an increase in temperature will be as follows:

$$d_g = \Delta T e_g a \qquad (1)$$

$$d_c = \Delta T e_c a \qquad (2)$$

where

d_g and d_c are the unrestrained radial displacement of the aggregate and mortar, respectively;

ΔT = Temperature change;

e_g and e_c are the thermal coefficients of expansion of the aggregate and mortar, respectively.

Then

$$q_g - q_c = \Delta T\, a (e_g - e_c) \qquad (3)$$

Actually the boundary of the aggregate and mortar will be between q_g and q_c, and is indicated by the dashed circle in Fig. 1. The restrained radial displacement of the aggregate is noted as x, and the restrained radial displacement of the mortar will then be equal to $q_g - q_c - x$.

According to Gerald Pickett, the radial displacement in the mortar at the boundary of the inclusion will be[1]

$$q_g - q_c - x = \frac{pa}{E_c} \left[\frac{1-U_c}{2} \cdot \frac{b^3 + 2a^3}{b^3 - a^3} + U_c \right] \qquad (4)$$

where

p = The unit pressure between the inner and outer spheres,

U_c = Poisson's ratio for the mortar, and

E_c = Young's modulus for the mortar.

Considering now the restrained volume change of the aggregate, an expression for x can be obtained in terms of the pressure p, radius, modulus of elasticity, and Poisson's ratio of the aggregate.

The restrained volume change can be expressed in the form

$$dV = 4\pi a^2 x \qquad (5)$$

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[1] Gerald Pickett, Effects of Aggregate upon Shrinkage and the Reversibility of Changes in Volume, Unpublished Report.

From the Bulk Modulus the unit volume change of the aggregate is

$$\frac{3(1-2U_g)p}{E_g} .$$

Therefore

$$4\pi a^2 x = \frac{3(1-2U_g)p}{E_g} \cdot \frac{4}{3}\pi a^3$$

$$x = \frac{(1-2U_g)pa}{E_g} \qquad (6)$$

where E_g and U_g are Young's modulus and Poisson's ratio, respectively, for the aggregate.

According to the theory of elasticity the tensile stress in the mortar is shown to be[2]

$$S_t = \frac{p}{2} \cdot \frac{b^3 + 2a^3}{b^3 - a^3} . \qquad (7)$$

Substituting equations 3, 6, and 7 into equation 4 and simplifying, the following expression for the tensile stress in the mortar will be

$$S_t = \frac{\Delta T \Delta C}{2\left[\left(\dfrac{b^3 - a^3}{b^3 + 2a^3}\right)\left(\dfrac{1-2U_g}{E_g} + \dfrac{U_c}{E_c}\right) + \dfrac{1-U_c}{2E_c}\right]}$$

where

E_g and U_g = Young's modulus and Poisson's ratio, respectively, for the aggregate;

E_c and U_c = Young's modulus and Poisson's ratio, respectively, for the mortar;

ΔC = Difference in thermal expansion of aggregate and mortar (per deg. F.)

ΔT = Temperature change in deg. F.

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[2] S. Timoshenko, Theory of Elasticity, p. 326.

Figure 2 indicates what effect the size and Young's modulus of the aggregate may produce on the tensile stress of the mortar, when the thermal expansion of the aggregate is considerably different from that of the mortar. In the calculation of the data for Fig. 2, the following conditions were assumed.

$$
\begin{aligned}
b &= 0.7'' \\
U_g &= 0.2 \\
U_c &= 0.15 \\
E_c &= 5 \times 10^6 \text{ p.s.i.} \\
\Delta T &= 60 \text{ deg. F.} \\
\Delta C &= 4 \times 10^{-6} \text{ / deg. F.}
\end{aligned}
$$

The assumptions are in good agreement with previous laboratory tests of the above properties and conditions. The stresses indicated in Fig. 2 are undoubtedly too large because very few concretes could sustain stresses of such magnitude. It should be pointed out that in the analysis, only one sphere was considered, and adjacent spheres would tend to offer considerable support to their neighbors.

With an increase in the modulus of elasticity of the aggregate one would expect higher stresses with the same thermal incompatibility of aggregate and mortar. This is evident in Fig. 2. It also indicates that lean mixtures might lead to serious trouble; i.e., as the cement content decreases, the thickness of the mortar surrounding the coarse aggregate also decreases.

It is to be noted in Fig. 2 that the stress is also plotted as a function of the ratio of a to b and changing the values of a and b will not affect the magnitude of the stress as long as their ratio remain constant.

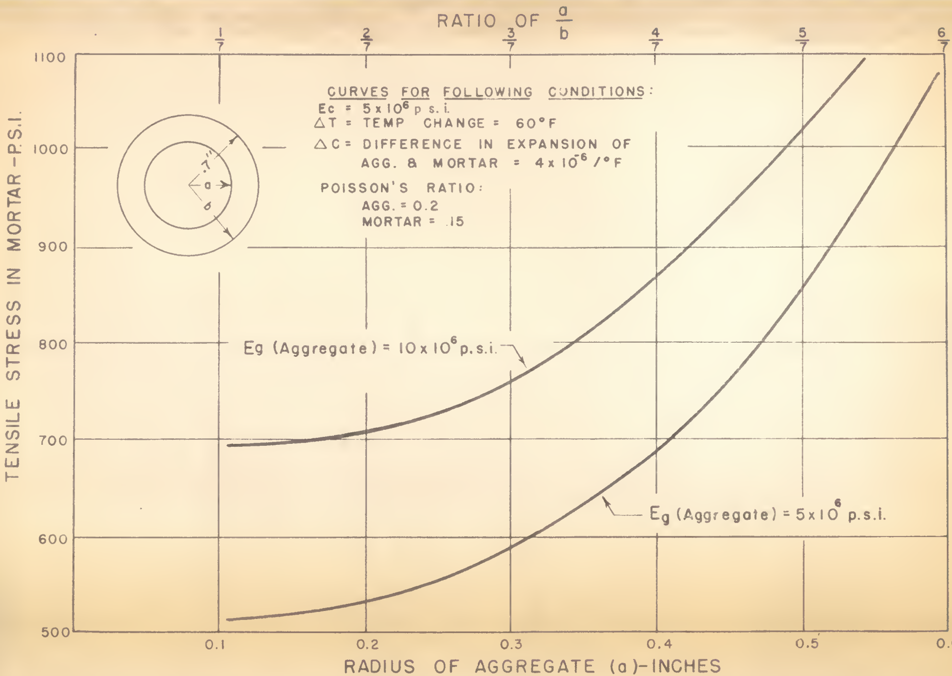

FIG. 2. TENSILE STRESS IN MORTAR MATRIX FROM THERMAL INCOMPATIBILITY

PART II. METHODS USED IN MEASURING THE THERMAL EXPANSION OF CONCRETE MORTAR AND COARSE AGGREGATE

Coarse Aggregate

The thermal expansion of coarse aggregates was determined by attaching SR-4 strain gages to small pieces of aggregate. By using a multiple switching unit, the thermal expansion of several specimens was determined quite easily with good results.

Procedure

One-inch cubes were prepared by the use of a small diamond saw. The surfaces were cleaned with carbon tetrachloride to remove all dirt and grease before attaching the strain gages. Two A-8 type gages were cemented to perpendicular faces as shown in Fig. 3. In this way any anisotropic behavior of the rock could be detected. After soldering leads to the gages, the specimens were placed in a water tight container and immersed in a constant temperature bath. The bath was lowered to about 36 degrees Fahrenheit by using chipped ice. After allowing the specimens to come to equilibrium, readings were taken with the Baldwin SR-4 Indicator. The bath was then heated to about 118 degrees Fahrenheit, and the readings were repeated. This cycle was repeated three times in order to obtain an average value of the thermal coefficient of expansion.

A thermocap relay, manufactured by the Niagara Electron Co., was used to control the bath at the higher temperature. A clip on the end of a wire from the relay is merely moved up and down

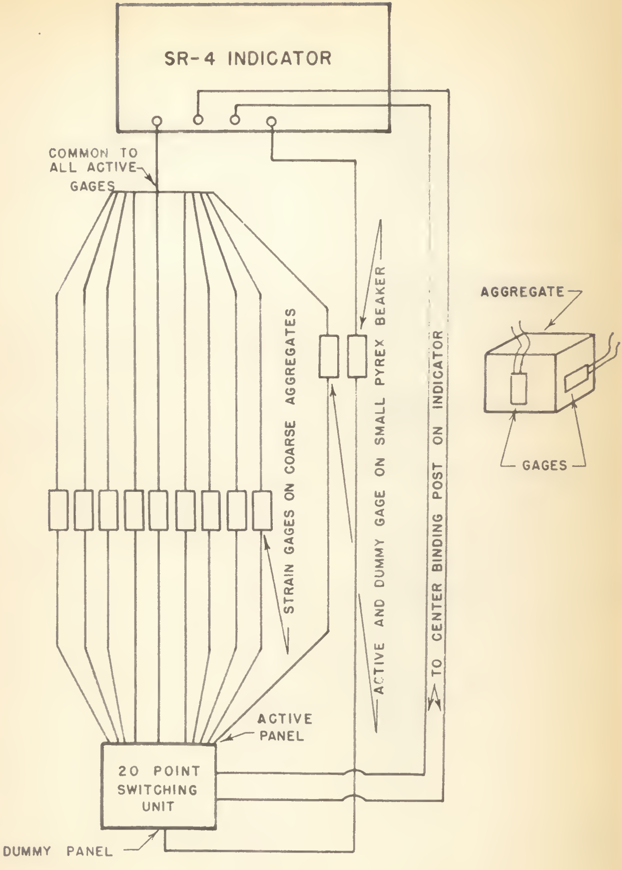

FIG. 3. WIRING DIAGRAM FOR MEASURING THERMAL EXPANSION OF COARSE AGGREGATE

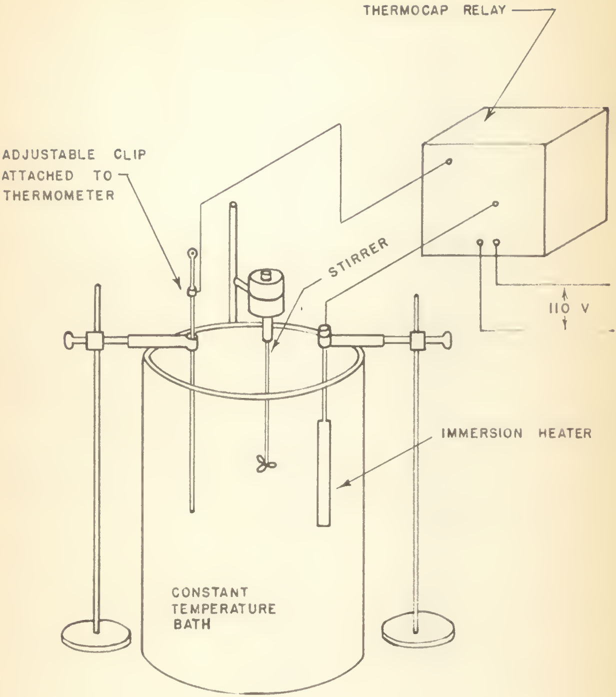

FIG. 4. SCHEMATIC DIAGRAM FOR CONTROL OF CONSTANT TEMPERATURE BATH

on a mercury thermometer to the desired temperature and an immersion heater is turned off and on by the relay as the mercury in the thermometer moves relative to the clip (causing a change in capacitance). With this control the bath temperature was controlled to two hundredths of a degree Fahrenheit.

In addition to the aggregate specimens placed in the container, an active and a dummy gage were mounted on a small Pyrex beaker. Provided there are no influences other than temperature effects, the difference between the active and the dummy gage, during a temperature change, should be zero. However, there may be changes in the resistance of the lead wires due to change in ambient conditions or in other components of the circuit. If there is a difference it is applied as a correction, being algebraically added or subtracted from the differences in readings for each specimen. In calculating the thermal expansion from the readings, the known thermal expansion of Pyrex (1.83×10^{-6} / deg. F.) is added to the change in readings of each specimen gage that occurred during the temperature change. Below is a sample calculation for the coefficient of expansion of Sioux City quartzite for cycle one. (See data in Table 1 for gage No. 1).

$$\text{Expansion / deg. F.} = \frac{(932 - 523 + 80 \times 1.83) \times 10^{-6}}{80} = 6.95 \times 10^{-6}$$

where the temperature change is 80 degrees Fahrenheit.

The thermal expansions for the coarse aggregates tested are tabulated in Table 1. The average value was obtained by averaging the six values obtained for the two gages for three cycles.

To determine the accuracy of the test a gage was cemented to a piece of 2S aluminum for which the coefficient of expansion (13.1×10^{-6} / deg. F.) was known. During each set of readings for the aggregate specimens, similar readings were obtained for the aluminum. Table 2 shows the values obtained by the SR-4 strain gage compared with those reported by other observers. This indicates that the method of using SR-4 strain gages gives results that are satisfactory for all practical purposes.

Measurement of Concrete Mortar

Two mortar bars, 1 x 1 x 11 inches, were molded for each mix with a water-cement ratio of 5.5 gallons per sack. Lone Star cement (type I) was used with the following sands:

Platte River
Kaw River
Blue River
Crushed Minnekahta limestone
Crushed Sioux City quartzite

Each of the above sands was carefully graded as follows:

Sieve size	Per cent retained
4	5
8	10
16	30
30	50
50	75
100	95

Gradation factor = 2.65

Table 1. Linear coefficient of expansion of coarse aggregates.

Coarse aggregates and cycle no.*	SR-4 indicator reading				Linear coefficient of expansion per degree Fahrenheit		Average linear coefficient of expansion per degree Fahrenheit
	Gage # 1		Gage # 2				
	Initial	Final	Initial	Final	Gage # 1	Gage # 2	
Sioux City quartzite							
One	5-523	5-932	5-158	5-568	6.95×10^{-6}	6.95×10^{-6}	6.9×10^{-6}
Two	5-498	5-918	5-150	5-560	6.9	6.8	
Three	5-490	5-920	5-150	5-570	7.08	6.95	
Lincoln sandstone							
One	5-1575	5-1830			5.0×10^{-6}	One	
Two	5-1575	5-1815			4.7	gage	4.9×10^{-6}
Three	5-1570	5-1820			4.9	only	
Minnekahta limestone							
One	5-340	5-385	5-1150	5-1180	2.4×10^{-6}	2.2×10^{-6}	
Two	5-365	5-398	5-1162	5-1185	2.2	2.09	2.2×10^{-6}
Three	5-368	5-412	5-1165	5-1195	2.37	2.2	
Maryland marble							
One	6-567	6-735	6-1644	6-1877	4.17×10^{-6}	5.07×10^{-6}	
Two	6-94	6-132	6-1176	6-1283	3.92	4.9	4.6×10^{-6}
Three	5-438	5-703	6-1075	6-1410	4.32	5.12	

* Temperature range for quartzite, sandstone, and limestone; cycle one (36 to 116 F), cycle two (35 to 117 F), cycle three (36 to 118 F). Temperature range for marble; cycle one (93 to 165 F), cycle two (93 to 164 F), cycle three (91 to 176 F).

Table 2. Linear coefficient of expansion of commercial aluminum (S-2).

Cycle no.	SR-4 indicator reading		Linear coefficient of expansion per degree Fahrenheit by SR-4	Other observers
	Initial	Final		
One	5-922	5-1785	12.6×10^{-6}	
Two	5-870	5-1778	12.9	13.1×10^{-6}
Three	5-860	5-1780	13.1	(68 to 212 F)
			Av. = 12.9×10^{-6}	

* Value taken from handbook printed by Aluminum Company of America.

Note: Temperature range; cycle one (36 to 116 F), cycle two (35 to 117 F), cycle three (36 to 118 F).

Each of the sands was combined with the cement in a ratio of 1 : 2.5 and 1 : 3.5 by weight. Specimens consisting of cement paste with and without fly ash were also molded so that the thermal expansion of cement paste could be determined. The bars were cured for seven days in the moist room followed by five days in laboratory air. At the end of this curing period the coefficient of expansion was determined by placing the specimens in the constant temperature bath used for coarse aggregates. The change in length was determined by a length comparator as shown in Fig. 5.

The expansion of the stainless steel plugs in the ends of the specimens was subtracted from the total change in length. This leaves the expansion of the mortar in a gage length of 10 inches. The coefficient of expansion of the steel plugs was assumed to be 6.5×10^{-6} / deg. F. The thermal expansion of the mortar specimens is then equal to the net change in length divided by 10 times the temperature change.

The thermal expansion for the various mortars and cement pastes are shown in Table 3. Cycles one through six indicate the thermal expansion for various degrees of moisture content and age. The cycles are described as follows:

Number of cycle	Age in days	Remarks
1	13	Readings were obtained after bars were cured 7 days in moist room and 5 days in laboratory air.
2	14	Readings were obtained after allowing bars to dry in laboratory air after immersion in bath for reading cycle one.
3	15	Readings were obtained following 24 hours soaking after cycle two.
4	30	Readings were obtained after allowing bars to dry in air from cycle three until 30 days of age.
5	31	Same as cycle two.
6	32	Same as cycle three.

No attempt was made to determine the per cent of saturation. The cycles described above should give various degrees of saturation and indicate the relative effect of moisture on the thermal expansion. Because the specimens were allowed to remain in the bath for a relatively short period (30 minutes), practically all of the variation in moisture content resulted from the disposition of specimens between cycles.

Conclusions

The data in Table 3 indicate that the thermal expansion of mortar is much lower than that of cement paste. They also indicate that the thermal expansion undergoes some variation with age, depending upon the amount of hydration. It is quite evident that the moisture content affects the thermal expansion of concrete more than age.

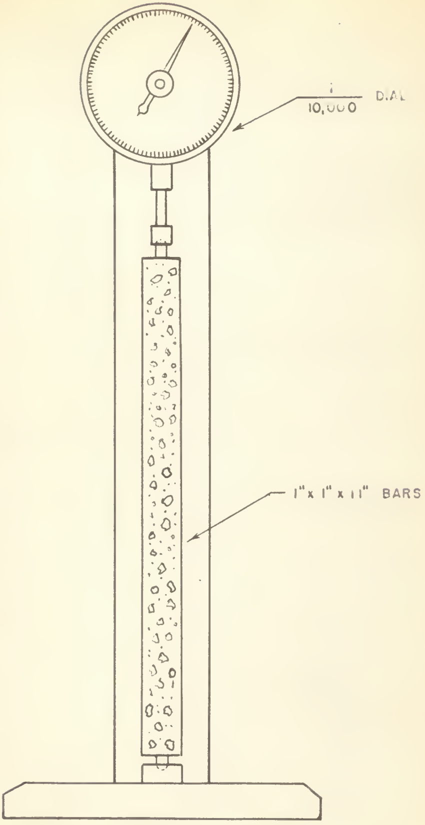

FIG. 5. LENGTH COMPARATOR USED IN MEASURING THERMAL EXPANSION OF MORTAR BARS

Table 3. Thermal expansion of mortar bars.

Identi-fica-tion	Fine aggregate	Mix by weight	Average thermal coefficient of expansion per degree Fahrenheit*					
			Cycle 1	Cycle 2	Cycle 3	Cycle 4	Cycle 5	Cycle 6
C	Cement paste	----	11.5×10^{-6}	9.9×10^{-6}	7.3×10^{-6}	11.6×10^{-6}	8.6×10^{-6}	----
CH	Cement + 30% Sub. Fly Ash	----	9.0	7.1	5.2	12.3	6.8	5.7×10^{-6}
PL	Platte River	1:2.5	6.4	5.6	5.2	7.0	5.2	5.2
PH	Platte River	1:3.5	6.3	5.2	5.1	6.4	5.0	5.1
BL	Blue River	1:2.5	6.3	5.9	5.2	7.5	5.4	5.2
BH	Blue River	1:3.5	6.3	5.5	5.0	6.8	5.4	5.3
KL	Kaw River	1:2.5	7.3	5.3	5.1	7.4	5.1	5.2
KH	Kaw River	1:3.5	6.9	5.7	5.3	6.9	5.7	5.2
KLH	Kaw River + 30% Sub Fly Ash	1:2.5	5.5	4.7	4.8	6.2	4.3	5.0
LL	Limestone	1:2.5	4.4	3.2	3.4	6.1	3.0	3.3
LH	Limestone	1:3.5	3.9	3.2	3.3	5.5	2.8	3.3
QL	Quartzite	1:2.5	7.2	6.3	6.4	9.3	5.6	6.1
QH	Quartzite	1:3.5	6.4	6.0	6.3	7.9	5.6	6.2

* Initial bath temperature 35 deg. F.
Final bath temperature 120 deg. F.

Substitution of fly ash reduces the thermal expansion; and the amount of sand, within reasonable limits, affects the coefficient of the mortar but very little.

Several of the cement past bars broke abruptly when undergoing a temperature change. This indicates that stresses in concrete might be sufficient to cause fracture when the concrete is undergoing a large temperature change.

PART III. THE EFFECT OF SIZE AND THERMAL EXPANSION ON THE DURABILITY OF CONCRETE AS DETERMINED BY FREEZING AND THAWING

Materials

For this phase of the study various mortars, for which the thermal expansion had been previously determined, were selected from the standpoint of varying degrees of incompatibility with the coarse aggregates to be used. The mortars selected contained the following fine aggregates:

Kaw River sand
Crushed Sioux City quartzite
Crushed Minnekahta limestone

Each of these sands was combined with each of the following coarse aggregates:

Sioux City quartzite
Minnekahta limestone
Lincoln sandstone

In order to study the effect of size, the coarse aggregates were graded into two different sizes. One size passed the 1-inch sieve and was retained on the 3/4-inch sieve; the other size passed the 1/2-inch sieve and was retained on the 3/8-inch sieve.

The total aggregate content was made up of 50 per cent fine aggregate and 50 per cent coarse aggregate. The grading for the fine aggregate was the same as shown in Part II of this text. Lone Star cement (type I) was used in all mixes.

Specimens

Two 3 x 4 x 16 inch beams were molded for each of the various combinations (18 different mixes) of fine and coarse aggregates of two sizes. The mixes were 1 : 5 by weight. These specimens were molded according to good laboratory procedure, with a water-cement ratio of 6 gallons per sack. An air content of 6 $\pm$ 0.5 per cent was used to eliminate the variable durability, as affected by the air content. According to H. F. Gonnerman, concretes with coarse aggregates having air contents of around 4 per cent show about as good resistance as those with higher air contents.[3]

The specimens were cured in the moist room for 7 days and then removed to laboratory air until 13 days of age. They were then soaked in tap water for 24 hours, immediately after which initial readings of weight, length, and dynamic modulus were recorded.

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[3] H. F. Gonnerman, "Tests of Concrete Containing Air-Entraining Portland Cement," Journal of the American Concrete Institute, 15:477, June, 1944.

Freezing and Thawing Cycles

All specimens were placed in the freezing and thawing unit immediately following the initial readings. The freezing and thawing cycles consisted of freezing at -20 deg. F. for 22 hours and thawing in a water bath at 75 deg. F. for 1 hour. The rate of temperature change (determined by a thermocouple) at the center of a 3 x 4 x 16 inch beam is shown in Fig. 6.

Deterioration from Freezing and Thawing

The best known criteria for determining the deterioration of concrete from freezing and thawing are per cent expansion and the loss in the modulus of elasticity. From data on resonant frequency, weight, and dimensions of the specimens, Young's modulus of elasticity can be determined from the following equation[4]

$$E = CWn^2$$

where

E = Young's modulus,

W = weight of the specimen,

n = a resonant frequency, and

C = a factor which depends upon the shape and size of specimen, the mode of vibration, and Poisson's ratio.

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[4] Gerald Pickett, "Equations for Computing Elastic Constants from Flexural and Torsional Resonant Frequencies of Vibration of Prisms and Cylinders," Research Laboratory of the Portland Cement Association, Bulletin 7, September, 1945.

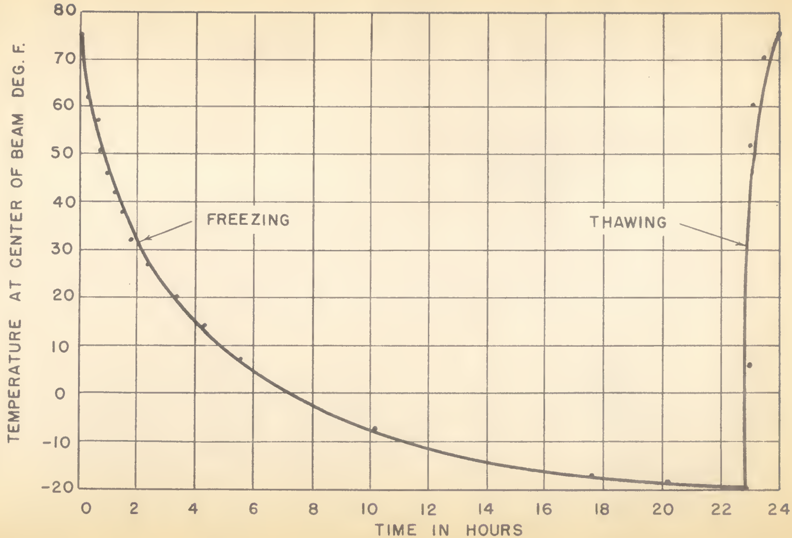

FIG. 6. TYPICAL FREEZING AND THAWING CYCLE

All data in this report are based upon the first mode of flexural vibration. The constant C for a 3 x 4 x 16 inch specimen vibrating in the 3 inch direction is equal to 0.1144. The difference in the thermal expansion of the mortar and aggregate, per cent air and size of coarse aggregate for the various mixes are shown in Table 4.

DISCUSSION

The values of the modulus of elasticity for the various concretes after 180 cycles of freezing and thawing are shown in Table 4. It is evident that very little deterioration has occurred for any of the concretes and in general all would be considered quite resistant to freezing and thawing. This high degree of resistance to freezing and thawing in undoubtedly due to the high air content.

The theoretical analysis indicated that the size of aggregate might have some effect on the durability of concrete where the thermal expansion of the aggregate was considerably different from that of the mortar. As shown in Table 4 the loss in modulus of the concretes containing 1 - 3/4 inch aggregate was consistently greater than that in the concretes containing 1/2 - 3/8 inch aggregate. The fact that the 1 - 3/4 inch aggregate was less durable in every case than the 1/2 - 3/8 inch aggregate seems highly significant even though the degree of durability may not be significant. It seems probable that with more cycles of freezing and thawing, the concretes with 1 - 3/4 inch aggregate may show an even greater loss in modulus than that of the concretes with 3/4 - 3/8 inch aggregate. The specimens are still under test. It should be pointed out that 3/4 inch aggregate is not very large and that possibly with aggregates as large as 1 or 2 inches, significant deterioration would have occurred. As evidence that larger aggregates should be given further consideration, some freezing and thawing data from the Kansas Highway Commission are tabulated below.

Per cent loss in Young's modulus for freezing and thawing of concrete with one and two inch aggregates

Identification	Average per cent loss per cycle of F & T	
	1" Aggregate	2" Aggregate
1M	2.41	6.39
2M	2.76	6.03
1MC	3.46	6.35
2MC	3.46	6.63
1OF	2.01	5.92
2OF	2.28	7.36

These were all non-air entraining concretes. Unfortunately the thermal coefficient of expansion of the aggregate and the air content of the concretes are not known. Certainly the soundness of the aggregate used in the above concretes is to be questioned. In view of the uncertainties it still seems high significant that the concrete with the two inch aggregate suffered from almost two to three times as much loss in modulus as the concrete with the one inch aggregate during freezing and thawing.

It is a well known fact that very lean mixtures are not very resistant to freezing and thawing. The author believes that this is due in part to the thin layer of mortar surrounding the coarse

Table 4. Percent loss in Young's modulus and percent expansion after 180 cycles of freezing and thawing.

Ident.	% air	Fine and coarse aggregate	Size of coarse aggregate	ΔC*	Young's modulus		Final modulus as percent of the initial (average)	Percent expansion	
					Initial x 10^6	After 180 cycles of F & T		Each beam	Average
TQCS-1	4.4	crushed quart-	3/8-1/2	3.9	5.82	6.07	104%	No expansion	
TQCS-2		zite and lime-			5.67	5.88		for any speci-	
TQCL-1	6.5	stone	1/2-3/4	3.9	4.73	4.63	96%	mens	
TQCL-2					5.10	4.83			
TLBS-1	7.5	crushed lime-	3/8-1/2	3.6	4.13	4.48	106%		
TLBS-2		stone and Sioux			4.23	4.43			
TLBL-1	6.0	quartzite	1/2-3/4	3.6	5.05	5.26	104%		
TLBL-2					4.97	5.17			
TKCS-1	5.5	Kaw river and	3/8-1/2	3.0	5.51	5.69	102%		
TKCS-2		limestone			5.74	5.77			
TKCL-1	5.5		1/2-3/4	3.0	5.49	5.36	98%		
TKCL-2					5.16	5.04			
TKBS-1	7.0	Kaw river and	3/8-1/2	1.7	5.37	5.67	105%		
TKBS-2		Sioux quartzite			4.99	5.16			
TKBL-1	6.0		1/2-3/4	1.7	5.22	5.25	100%		
TKBL-2					5.72	5.70			
TLAS-1	5.5	Crushed lime-	3/8-1/2	1.6	4.70	4.88	104%		
TLAS-2		stone and sand-			4.63	4.86			
TLAL-1	5.6	stone	1/2-3/4	1.6	4.86	4.89	102%		
TLAL-2					4.71	4.88			

Table 4. (Concl.)

Ident.	% air	Fine and coarse aggregate	Size of coarse aggregate	ΔC*	Young's modulus		Final modulus as percent of the initial (average)	Percent expansion	
					Initial x 10^6	After 180 cycles of F & T		Each beam	Average
TQAS-1	5.5	Crushed Sioux quartzite and sandstone	3/8-1/2	1.2	5.01	5.21	114%		
TQAS-2					5.33	5.58			
TQAL-1	6.5		1/2-3/4	1.2	5.08	5.30	103%		
TQAL-2					5.32	5.40			
TLCS-1	5.5	Crushed lime-stone and limestone	3/8-1/2	1.1	4.94	5.27	106%		
TLCS-2					4.61	4.87			
TLCL-1	8.5		1/2-3/4	1.1	4.74	4.93	104%		
TLCL-2					4.61	4.79			
TQBS-1	10.0	Crushed quart-zite and Sioux quartzite	3/8-1/2	0.8	4.53	4.99	110%		
TQBS-2					4.41	4.76			
TQBL-1	6.0		1/2-3/4	0.8	5.54	5.82	106%		
TQBL-2					5.42	5.76			
TKAS-1	7.0	Kaw river and sandstone	3/8-1/2	0.3	4.82	4.97	102%		
TKAS-2					4.72	4.74			
TKAL-1	5.5		1/2-3/4	0.3	4.92	4.90	100%		
TKAL-2					4.71	4.72			

* Difference in the thermal expansion of coarse aggregate and mortar.

aggregate. It was shown in Fig. 2 that the ratio of the radius of the inclusion to that of the surrounding medium affect the stresses which result from thermal incompatibility.

The theoretical analysis also indicated that the magnitude of the stress developed around an inclusion depends to a certain degree upon the modulus of elasticity of the material of the inclusion. This variable has been neglected in this study. The assumption made in the theoretical analysis as to the aggregate being an isolated spherical inclusion is of course in disagreement with the actual conditions within a body of concrete. However, within certain limitations the analysis shows that the study of thermal incompatibility involves not only the thermal expansion of aggregates but also their size and modulus of elasticity.

CONCLUSIONS

Since concrete is highly unpredictable, it is difficult to draw any definite conclusions from such a limited amount of data as that presented in this manuscript. However, from the data presented the following general conclusions can be drawn and these may serve as a guide for further research in the study of thermal incompatibility.

(a) The thermal expansion of mortar is quite variable, depending upon the degree of saturation.

(b) Air entrainment increases the resistance to freezing and thawing and would be beneficial when the thermal expansion of the aggregate is considerably different from that of the mortar.

(c) Coarse aggregates with undesirable coefficients will cause little trouble as long as the maximum size does not exceed 3/4 inch.

RECOMMENDATIONS FOR FURTHER STUDY

The author realizes the fact that there is a large amount of research possible in the study of thermal incompatibility, and that only a part was covered in this study. From the experience gained in this study the author feels that the following problems should be given further consideration.

(a) The effect of thermal incompatibility when using coarse aggregates with a maximum size of two inches.

(b) The effect of thermal incompatibility with concretes of lower air contents than those used in this study.

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THE EFFECT OF SIZE AND THERMAL EXPANSION
OF AGGREGATES ON THE DURABILITY OF CONCRETE

by

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The deterioration of concrete, exposed to the elements of nature, has been attributed to many factors related to cement-aggregate reaction and freezing and thawing. One of the most recent causes related to freezing and thawing is deterioration from aggregates having a different thermal coefficient of expansion than that of the cement paste. This is referred to as thermal incompatibility.

The author made a theoretical analysis of the stresses that might result in a matrix surrounding a spherical body when the thermal expansion of the matrix is different from that of the inclusion. This study indicated that the magnitude of the stresses depends upon the thermal expansion, Poisson's ratio, Young's modulus, and the size of the inclusion. Using the theoretical analysis as a guide, a study was made to determine the effect of the thermal incompatibility and size of aggregate upon the durability of concrete.

The thermal coefficients of expansion of several coarse aggregates were determined by attaching SR-4 strain gages to each of the aggregates. The thermal expansions of 1 x 1 x 11 inch mortar bars, containing various fine aggregates were determined by a length comparator.

Two 3 x 4 x 16 inch specimens were molded for each of the various combinations of fine and coarse aggregates. In order to study the effect of aggregate size, the coarse aggregates were graded into two different sizes. One size passed the one inch sieve and was retained on the 3/4 inch sieve; the other passed the 1/2 inch sieve and was retained on the 3/8 inch sieve.

The per cent expansion and loss in dynamic modulus were determined for the specimens after 180 cycles of freezing and thawing. Very little deterioration occurred for any of the concretes which in general would be considered quite resistant to freezing and thawing. Therefore no significant difference in durability was apparent for the concretes with varying degrees of thermal incompatibility. The loss in the modulus of the concretes containing 1 - 3/4 inch aggregate, were consistently greater than those concretes containing 1/2 - 3/8 inch aggregate but this difference in durability was slight and could not be considered as significant. The fact that the 1 - 3/4 inch aggregate was consistently less durable than the 1/2 - 3/8 inch aggregate should not be overlooked and indicates that there is some merit in the theoretical analysis.

Freezing and thawing data from the Kansas Highway Commission showed that concretes with two inch aggregate were much less durable to freezing and thawing than those concretes with one inch aggregate. This lends evidence that the deterioration from thermal incompatibility should be given further consideration by studying the effect of 2 inch aggregates with various thermal coefficients of expansion.